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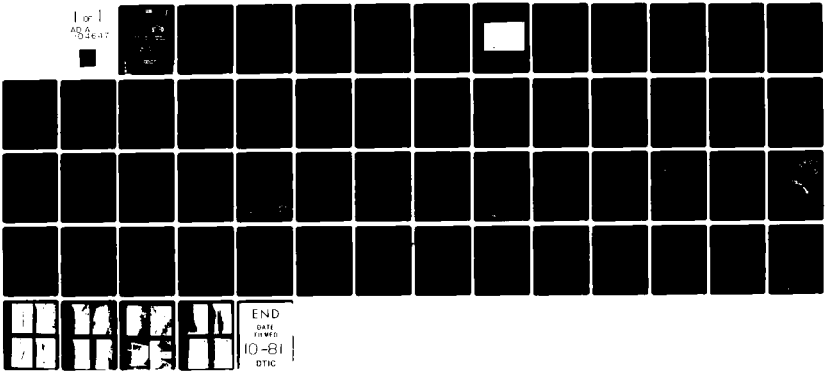

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MITCHELL LAKE DAM
DENT COUNTY, MISSOURI
MO 30268

PHASE 1 INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

United States Army
Corps of Engineers
...Serving the Army
...Serving the Nation

St. Louis District

PREPARED BY: U.S. ARMY ENGINEER DISTRICT, ST. LOUIS

FOR: STATE OF MISSOURI

DECEMBER 1979

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SECURITY CLASSIFICATION OF THIS PAGE *(When Data Entered)*

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Dam Safety, Lake, Dam Inspection, Private Dams

20. ABSTRACT *(Continue on reverse side if necessary and identify by block number)*

This report was prepared under the National Program of Inspection of Non-Federal Dams. This report assesses the general condition of the dam with respect to safety, based on available data and on visual inspection, to determine if the dam poses hazards to human life or property.

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DEPARTMENT OF THE ARMY
ST. LOUIS DISTRICT, CORPS OF ENGINEERS
210 NORTH 12TH STREET
ST. LOUIS, MISSOURI 63101

IN REPLY REFER TO

SUBJECT: Mitchell Lake Dam Phase I Inspection Report

This report presents the results of field inspection and evaluation of the Mitchell Lake dam:

It was prepared under the National Program of Inspection of Non-Federal Dams

This dam has been classified as unsafe, non-emergency by the St. Louis District as a result of the application of the following criteria:

1) Spillway will not pass 50 percent of the Probable Maximum Flood
2) Overtopping could result in dam failure
3) Dam failure significantly increses the hazard to loss of life downstream

SUBMITTED BY: SIGNED
Chief, Engineering Division

7 APR 1980
Date

APPROVED BY: SIGNED
Colonel, CE, District Engineer

8 APR 1980
Date

MITCHELL LAKE DAM

DENT COUNTY, MISSOURI

MISSOURI INVENTORY NO. 30268

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

Prepared By

Anderson Engineering, Inc., Springfield, Missouri
Hanson Engineers, Inc., Springfield, Illinois

Under Direction Of

St. Louis District, Corps of Engineers

For

Governor of Missouri

MARCH 1980

PHASE I REPORT
NATIONAL DAM SAFETY PROGRAM

Name of Dam:	Mitchell Lake Dam
State Located:	Missouri
County Located:	Dent
Stream:	Unnamed Tributary of Meramec River
Date of Inspection:	August 15, 1979

Mitchell Lake Dam was inspected by an interdisciplinary team of engineers from Anderson Engineering, Inc. of Springfield, Missouri and Hanson Engineers, Inc. of Springfield, Illinois. The purpose of the inspection was to make an assessment of the general condition of the dam with respect to safety, based upon available data and visual inspection, in order to determine if the dam poses hazards to human life or property.

The guidelines used in the assessment were furnished by the Department of the Army, Office of the Chief of Engineers, and they have been developed with the help of several Federal and State agencies, professional engineering organizations, and private engineers. Based on these guidelines, the St. Louis District, Corps of Engineers has determined that this dam is in the high hazard potential classification, which means that loss of life and appreciable property loss could occur if the dam fails. The estimated damage zone extends approximately three miles downstream of the dam. Located within this zone are three dwellings. The dam is in the small size classification, since it is greater than 25 ft high but less than 40 ft high, and the maximum storage capacity is greater than 50 ac-ft but less than 1000 ac-ft.

Our inspection and evaluation indicates that the combined spillways do not meet the criteria set forth in the guidelines for a dam having the above size and hazard potential. The combined spillways will pass 26 percent of the Probable Maximum Flood without overtopping. The Probable Maximum Flood is defined as the flood discharge that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region. The guidelines require that a dam of small size with a high downstream hazard potential pass 50 to 100 percent of the PMF. Considering the small size of the dam, the low storage capacity of the reservoir, the large floodplain downstream, and the flood storage capacity of the three lakes located immediately upstream, 50 percent

of the PMF has been determined to be the appropriate spillway design flood. The 100-year frequency flood will not overtop the dam. The 100-year flood is one that has a 1 percent chance of being exceeded or equaled in any given year.

Deficiencies visually observed by the inspection team were: (1) some minor wave erosion on the upstream face of the dam; (2) possible seepage areas at the toe of the embankment (Station 3+25) and at the base of the right abutment; (3) some very minor sloughing on the downstream embankment face; and (4) some brush growth on the embankment face. Another deficiency was the lack of seepage and stability analysis records.

It is recommended that the owners take the necessary action in the near future to correct the deficiencies reported herein. A detailed discussion of these deficiencies is included in the following report.

Jack Healy, P.E. (HEI)

Steve Brady, P.E., (AEI)

Nelson Morales, P.E. (HEI)

Tom Beckley, P.E. (AEI)

Dan Kerns, E.I.T. (HEI)

AERIAL VIEW OF LAKE AND DAM

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
MITCHELL LAKE DAM - ID No. 30268

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SECTION 1 - PROJECT INFORMATION

1.1 GENERAL:

A. Authority:

The National Dam Inspection Act, Public Law 92-367, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a program of safety inspection of dams throughout the United States. Pursuant to the above, the St. Louis District, Corps of Engineers, District Engineer directed that a safety inspection be made of Mitchell Lake Dam in Dent County, Missouri.

B. Purpose of Inspection:

The purpose of the inspection was to make an assessment of the general condition of the dam with respect to safety, based upon available data and a visual inspection in order to determine if the dam poses hazards to human life or property.

C. Evaluation Criteria:

Criteria used to evaluate the dam were furnished by the Department of the Army, Office of the Chief of Engineers, "Recommended Guidelines for Safety Inspection of Dams, Appendix D." These guidelines were developed with the help of several federal agencies and many state agencies, professional engineering organizations, and private engineers.

1.2 DESCRIPTION OF PROJECT:

A. Description of Dam and Appurtenances:

Mitchell Lake Dam is an earth fill structure approximately 35 ft high and 600 ft long at the crest. The appurtenant works consist of a primary spillway consisting of two, 20 in. diameter steel pipes located near the north end of the dam, a grass-covered emergency spillway located at the south abutment and an 8 in. diameter steel drawdown pipe and associated valve located at the primary spillway outlet. Sheet 3 of Appendix A shows a plan profile and typical section of the embankment.

B. Location:

The dam is located in the north central part of Dent County, Missouri on a tributary of Meramec River. The dam and lake are within the Stone Hill, Missouri 15 minute quadrangle sheet (Section 4, T34N, R5W - latitude 37° 41.5'; longitude 91° 29.5'). Sheet 2 of Appendix A shows the general vicinity.

C. Size Classification:

With an embankment height of 35 ft and a maximum storage capacity of approximately 526 acre-ft, the dam is in the small size category.

D. Hazard Classification:

The St. Louis District, Corps of Engineers has classified this dam as a high hazard dam. The estimated damage zone extends approximately three miles downstream of the dam. Located within this zone are three dwellings.

E. Ownership:

The dam is owned by Dr. Roy Mitchell. The owner's address is Route 7, Box 36, Salem, Missouri 65560.

F. Purpose of Dam:

The dam was constructed primarily for recreational purposes, although some flood protection is also provided.

G. Design and Construction History:

No design information or plans were available. The dam was constructed in 1960 by Mr. E. J. Mooney, Mr. Clark Wines and Mr. Noah Wisdom. Information from the present owner indicates that the construction was observed by Mr. Marvin Holland of the Missouri Conservation Commission. Material for construction of the embankment was obtained from the north abutment hillside. Mr. Holland indicated that a key trench was utilized in construction. The earth swale spillway located at the north abutment was washed out by a flood in 1961. This area was repaired in 1962, and a new spillway was constructed at the south abutment.

The present owner acquired the dam in 1968. The primary spillway pipes were installed in about 1970. In addition, the emergency spillway was widened, and material was placed along the downstream toe of the dam.

H. Normal Operating Procedures:

The owner indicated that the lake is drawn down 5 to 6 ft every winter to control aquatic vegetation.

1.3 PERTINENT DATA:

Pertinent data about the dam, appurtenant works, and reservoir are presented in the following paragraphs. Sheet 3 of Appendix A presents a plan, profile and typical section of the embankment.

A. Drainage Area:

The drainage area for this dam, as obtained from the U.S.G.S. quad sheet, is approximately 692 acres.

B. Discharge at Dam Site:

(1) All discharge at the dam site is through uncontrolled spillways.

(2) Estimated Total Spillway Capacity at Maximum Pool (Top of Dam - El. 1133.2): 2320 cfs

(3) Estimated Capacity of Primary Spillway: 54 cfs

(4) Estimated Experienced Maximum Flood at Dam Site: Unknown

(5) Diversion Tunnel Low Pool Outlet at Pool Elevation: Not Applicable

(6) Diversion Tunnel Outlet at Pool Elevation: Not Applicable

(7) Gated Spillway Capacity at Pool Elevation: Not Applicable

(8) Gated Spillway Capacity at Maximum Pool Elevation: Not Applicable

C. Elevations:

All elevations are consistent with an assumed mean sea level elevation of 1128.0 for the crest of the steel weir for the overflow spillway pipes (estimated from quadrangle map).

(1) Top of Dam: 1133.2 (Low Point); 1135.5 (High Point)

(2) Principal Spillway Crest: 1128.0

(3) Emergency Spillway Crest: 1130.7

(4) Principal Outlet Pipe Invert: 1102.1

(5) Streambed at Centerline of Dam: 1100.5

(6) Pool on Date of Inspection: 1123.7

(7) Apparent High Water Mark: None Apparent

(8) Maximum Tailwater: Unknown

(9) Upstream Portal Invert Diversion Tunnel: Not Applicable

(10) Downstream Portal Invert Diversion Tunnel: Not Applicable

D. Reservoir Lengths:

(1) At Top of Dam: 3200 ft

(2) At Principal Spillway Crest: 2500 ft

(3) At Emergency Spillway Crest: 2770 ft

E. Storage Capacities:

(1) At Principal Spillway Crest: 269 ac-ft

(2) At Top of Dam: 526 ac-ft

(3) At Emergency Spillway Crest: 369 ac-ft

F. Reservoir Surface Areas:

(1) At Principal Spillway Crest: 29 ac.

(2) At Top of Dam: 44 ac.

(3) At Emergency Spillway Crest: 35 ac.

G. Dam:

(1) Type: Earth

(2) Length at Crest: 600 ft

(3) Height: 35 ft

(4) Top Width: 16 ft

(5) Side Slopes: Upstream Irregular; Downstream Irregular (see Sheet 3, Appendix A)

(6) Zoning: Apparently homogeneous

(7) Impervious Core: Unknown

(8) Cutoff: Key trench 8 ft to 10 ft deep (from Marvin Holland)

(9) Grout Curtain: Unknown

H. Diversion and Regulating Tunnel:

(1) Type: Not Applicable

(2) Length: Not Applicable

(3) Closure: Not Applicable

(4) Access: Not Applicable

(5) Regulating Facilities: Not Applicable

I. Spillway:

I.1 Principal Spillway:

(1) Location: Near north end of dam

(2) Type: Two 20 in. diameter steel pipes enclosed by steel weir

I.2 Emergency Spillway:

(1) Location: South abutment

(2) Type: Earth swale

J. Regulating Outlets:

An 8 in. diameter steel pipe is provided for dewatering the lake. Reportedly, the pipe will lower the lake at a rate of about 4 in. per day. The intake structure is located about 50 ft to 75 ft into the lake directly out from the primary spillway inlet. The intake consists of a perforated standpipe which allows complete drainage of the reservoir. The outlet and valve for the drawdown pipe are located adjacent to the primary spillway outlet.

SECTION 2 - ENGINEERING DATA

2.1 DESIGN:

Although it was reported that Mr. Marvin Holland of the Missouri Conservation Commission observed the construction and provided some construction assistance for Mitchell Lake Dam, no design computations or reports could be located. No documentation of construction inspection records have been obtained. To our knowledge, there are no documented maintenance data.

A. Surveys:

No information regarding pre-construction surveys was able to be obtained. Sheet 3 of Appendix A presents a plan, profile and cross section of the dam from survey data obtained during the site inspection. The crest of the steel overflow weir for the primary spillway pipes was used as a site datum of assumed elevation 1128.0 MSL (see Sheet 3, Appendix A). The elevation of this site datum was estimated using the U.S.G.S. topographic map.

B. Geology and Subsurface Materials:

The site is located in the central portion of the Ozarks geologic region of Missouri. The Ozarks are characterized topographically by hills, plateaus and deep valleys. The most common bedrock types are dolomite, sandstone and chert. The "Geologic Map of Missouri" indicates that the bedrock in the site area consists primarily of the Gasconade formation of the Canadian Series in the Ordovician System. The Gasconade formation is predominantly a light brownish-gray, cherty dolomite. In the central Ozarks region, the average thickness of the Gasconade is 300 ft. Caves and springs are common in this formation.

The publication "Caves of Missouri" indicates that nine known caves exist in Dent County. Of the four caves within 10 miles of the site, three are clustered about 4 miles northeast of the site, and the fourth cave is about 4 miles southeast of the site.

The "Geologic Map of Missouri" indicates a normal fault passing about 11 miles northwest of the site in an east-west direction. The Missouri Geological Survey

has indicated that the faults in this area are generally considered to be inactive and have been for several hundred million years.

The soils in the area of the dam are of the Nixa-Clarksville-Lebanon-Hobson soil association. These soils have developed from cherty dolomite, limestone and sandstone. The thickness of loessial deposits in upland areas may range from 2.5 ft to 5.0 ft. The published "Soil Survey of Dent County" indicates that the soils adjacent to the reservoir are the Coulstone and Clarksville cherty soils. These soils consist of a yellowish-red very cherty silty clay loam.

C. Foundation and Embankment Design:

No foundation and embankment design information was available. Seepage and stability analyses as required by the guidelines were not able to be obtained. Marvin Holland indicated that a key trench (cutoff) was built under the embankment. The key trench was reportedly about 8 ft to 10 ft deep and as wide as a bulldozer blade. No internal drainage features are known to exist. No construction inspection test results have been obtained.

D. Hydrology and Hydraulics:

No hydrologic and hydraulic design computations for Mitchell Lake Dam were available. Based on a field check of spillway dimensions and embankment elevations, and a check of the drainage area on the U.S.G.S. quadrangle sheet, hydrologic analysis using U.S. Corps of Engineers guidelines were performed and appear in Appendix C, Sheets 1 to 8. It was concluded that the structure will pass 26 percent of the Probable Maximum Flood without overtopping. The 100-year frequency flood will not overtop the dam.

E. Structure:

The appurtenant structures include the two primary spillway pipes, intake structure and drawdown pipes. In addition, a concrete seepage collar is provided for the spillway pipes just downstream of the intake structure. It is reported that bentonite was used in the soil around the concrete collar. No design information concerning these structures is available.

2.2 CONSTRUCTION:

No construction inspection data have been obtained.

2.3 OPERATION AND MAINTENANCE:

Normal flows are passed by the uncontrolled steel pipe primary spillway, whereas a grassed emergency spillway will come into operation for major floods. The 8 in. diameter drawdown pipe is reportedly used every year to lower the lake 5 to 6 ft during the fall and winter. This is done in an attempt to control aquatic vegetation around the perimeter of the lake. The owner indicated that the embankment face is never mowed.

2.4 EVALUATION:

A. Availability:

No engineering data, seepage or stability analyses, or construction test data were available.

B. Adequacy:

The engineering data available were inadequate to make a detailed assessment of the design, construction, and operation of this structure. Seepage and stability analyses comparable to the requirements of the "Recommended Guidelines for Safety Inspection of Dams" were not available, which is considered a deficiency. These seepage and stability analyses should be performed for appropriate loading conditions (including earthquake loads) and made a matter of record.

C. Validity:

To our knowledge, no valid engineering data on the design or construction of the embankment are available.

SECTION 3 - VISUAL INSPECTION

3.1 FINDINGS:

A. General:

The field inspection was made on August 15, 1979. The inspection team consisted of personnel from Anderson Engineering, Inc. of Springfield, Missouri and Hanson Engineers, Inc. of Springfield, Illinois. The team members were:

John Healy - Hanson Engineers, Inc. (Geotechnical Engineer)
Steve Brady - Anderson Engineering, Inc. (Civil Engineer)
Tom Beckley - Anderson Engineering, Inc. (Civil Engineer)
Nelson Morales - Hanson Engineers, Inc. (Hydraulic Engineer)
Dan Kerns - Hanson Engineers, Inc. (Geotechnical Engineer)

B. Dam:

The dam appears to be generally in good condition. Some minor wave erosion was noted on the upstream face. No wave protection is provided for the upstream face of the dam. Some very minor sloughing was observed on the downstream face of the dam. Possible seepage areas were noted at the toe of the dam (Station 3+25) and at the base of the right abutment. These areas were soft and wet, although no seepage flows were observed. Although these wet areas could be attributed to poor drainage, staining of the standing water would indicate that the marshy areas are probably created by seepage.

The crest of the dam slopes downward to the north (see Sheet 3, Appendix A). This sloping results in about a 3 ft difference in crest elevation across its length. The horizontal alignment of the embankment is good. No surface cracks or unusual movements were noted. Some small brush growth was present on the embankment face. The owner reported that the dam is not mowed.

Shallow auger probes into the embankment indicated the dam to consist of a reddish brown silty clay with rock fragments. Information from Marvin Holland indicated that borrow material for construction of the dam was obtained from the north abutment hillside.

C. Appurtenant Structures:

C.1 Primary Spillway:

The area around the intake to the primary spillway was clear. The outlet channel below the plunge pool is overgrown with brush and trees (see Photos 10 and 12). The 20 in. diameter steel pipes appeared to be in good condition.

C.2 Emergency Spillway:

The grass-covered earth emergency spillway appeared to be in good condition. The approach area to the emergency spillway was clear. No erosion or debris was noted in the emergency spillway.

C.3 Drawdown Pipe:

The 8 in. diameter steel drawdown pipe was operating during the inspection. Neither the outlet nor the valve was visible, as they were submerged in the plunge pool (see Photo 11).

D. Reservoir:

The watershed is generally wooded and grass-covered with no agricultural activity. The slopes adjacent to the reservoir are moderate to steep, and no sloughing or serious erosion was noted. The water level was several feet below normal pool during the inspection, and some sedimentation was observed in the upper reaches of the lake.

E. Downstream Channel:

The downstream channel of the primary spillway is heavily wooded. The emergency spillway downstream channel is fairly clear for several hundred yards before entering a wooded area.

3.2 EVALUATION:

Weeds and brush on the dam encourage animal burrowing. The erosion and sloughing on the upstream face and portions of the downstream face indicates that the erosion resistance is inadequate in these areas. The heavily wooded area at

the primary spillway outlet could restrict flood flows. Existing seepage and marshy areas, if left uncontrolled, may adversely affect the stability of the dam. These deficiencies should be corrected under the direction of an engineer experienced in the design and construction of dams.

Because the valve of the lake drain is located on the downstream side of the dam, the full head of water impounded by the dam is acting entirely through the dam. The area around the lake drain outlet should be periodically inspected for seepage which might indicate a leak or rupture of the drain pipe and could eventually initiate a piping failure through the embankment.

Photographs of the dam, appurtenant structures, and the reservoir are presented in Appendix D.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 PROCEDURES:

The owner reported that the lake level is lowered several feet every year to control aquatic vegetation. The pool is normally controlled by rainfall, runoff, evaporation, seepage, and the capacities of the uncontrolled spillways.

4.2 MAINTENANCE OF DAM:

Weed and brush growth on the embankment face and some sloughing on the upstream face indicate that the dam has not recently been maintained.

4.3 MAINTENANCE OF OPERATING FACILITIES:

No regular maintenance program for the dewatering facility is known to exist.

4.4 DESCRIPTION OF ANY WARNING SYSTEM IN EFFECT:

The inspection team is unaware of any existing warning system for this dam.

4.5 EVALUATION:

The sloughing on the upstream face of the dam, the weed and brush growth on the embankment face, minor sloughing on the downstream face of the embankment, and the wet, soft areas at the toe of the dam are deficiencies which should be corrected. However, these should only be accomplished under the direction of an experienced engineer to avoid creating an unsafe condition.

SECTION 5 - HYDRAULIC/HYDROLOGIC

5.1 EVALUATION OF FEATURES:

A. & B. Design and Experience Data:

The hydraulic and hydrologic analyses were based on: (1) a field survey of spillway dimensions and embankment elevations; and (2) an estimate of the pool and drainage areas from the U.S.G.S. quad sheet. The owner indicated that the emergency spillway has not been used since the primary spillway pipes were installed in about 1970. The highest water over the primary spillway was reportedly about 10 in. above normal pool. Our hydrologic and hydraulic analyses using U. S. Army Corps of Engineers guidelines appear in Appendix C.

C. Visual Observations:

The approach channels to the primary and emergency spillways are clear. The primary spillway discharge channel is heavily wooded.

The spillway channels are well separated from the embankment, and spillway releases would not be expected to endanger the dam.

D. Overtopping Potential:

Based on the hydrologic and hydraulic analysis presented in Appendix C, the combined spillways will pass 26 percent of the Probable Maximum Flood. The Probable Maximum Flood is defined as the flood discharge that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region. The recommended guidelines from the Department of the Army, Office of the Chief of Engineers, require that this structure (small size with high downstream hazard potential) pass 50 percent to 100 percent of the PMF, without overtopping. Considering the small size of the dam, the low storage capacity of the reservoir, the large floodplain downstream, and the flood storage capacity of the three lakes located immediately upstream, 50 percent of the PMF has been determined to be the appropriate spillway design flood. The structure will pass a 100-year frequency flood without overtopping.

The routing of 50 percent of the PMF through the spillways and dam indicates that the dam will be overtopped by 1.39 ft at elevation 1136.39. The duration of the overtopping will be 1.42 hours, and the maximum outflow will be 6892 cfs. The maximum discharge capacity of the spillways is 2320 cfs. Overtopping of an earthen embankment could cause serious erosion and could possibly lead to failure of the structure. However, the materials which comprise the dam are not considered highly erodible, and the duration of overtopping is not excessive.

SECTION 6 - STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY:

A. Visual Observations:

Observed features which could adversely affect the structural stability of this dam are discussed in Sections 3.1B and 3.2.

B. Design and Construction Data:

No design and construction data for the foundation and embankment were available. Seepage and stability analyses comparable to the requirements of the guidelines were not available, which constitutes a deficiency which should be rectified.

C. Operating Records:

No operating records have been obtained.

D. Post-Construction Changes:

Shortly after completion in 1961, the earth swale spillway at the north abutment was washed out. This area was filled in and repaired in 1962, and a new spillway was constructed at the south abutment. The primary spillway pipes were installed in about 1970. In addition, the emergency spillway was widened, and material was placed at the downstream toe to widen it at the base.

E. Seismic Stability:

The structure is located in seismic zone 1. An earthquake of this magnitude would not generally be expected to cause severe structural damage to a well constructed earth dam of this size. However, it is recommended that the prescribed seismic loading for this zone be applied in stability analyses performed for this dam.

SECTION 7 - ASSESSMENT/REMEDIAL MEASURES

7.1 DAM ASSESSMENT:

This Phase I inspection and evaluation should not be considered as being comprehensive since the scope of work contracted for is far less detailed than would be required for an in-depth evaluation of dams. Latent deficiencies, which might be detected by a totally comprehensive investigation, could exist.

A. Safety:

The embankment is generally in good condition. Several items were noted during the visual inspection which should be investigated further, corrected or controlled. These items are: (1) minor wave erosion on the upstream face of the dam; (2) possible seepage area at the toe of the embankment (Station 3+25) and at the base of the right abutment; (3) some very minor sloughing on the downstream embankment face; and (4) some weed and brush growth on the embankment face.

Another deficiency is the lack of seepage and stability analysis records.

The dam will be overtopped by flows in excess of 26 percent of the Probable Maximum Flood. Overtopping of an earthen embankment could cause serious erosion and could possibly lead to failure of the structure.

B. Adequacy of Information:

The conclusions in this report were based on the performance history as related by the owner, and visual observation of external conditions. The inspection team considers that these data are sufficient to support the conclusions herein. Seepage and stability analyses comparable to the "Recommended Guidelines for Safety Inspection of Dams" were not available, which is considered a deficiency.

C. Urgency:

The remedial measures recommended in paragraph 7.2 should be accomplished in the near future. If the deficiencies listed in paragraph A are not corrected, and if good maintenance is not provided, the embankment condition will deteriorate and possibly could become serious in the future. The item recommended in paragraph 7.2A should be pursued without undue delay.

D. Necessity for Phase II:

Based on the result of the Phase 1 inspection, no Phase II inspection is recommended.

E. Seismic Stability:

The structure is located in seismic zone 1. An earthquake of this magnitude would not generally be expected to cause severe structural damage to a well constructed earth dam of this size. However, it is recommended that the prescribed seismic loading for this zone be applied in any stability analyses performed for this dam.

7.2 REMEDIAL MEASURES:

The following remedial measures and maintenance procedures are recommended. All remedial measures should be performed under the guidance of a professional engineer experienced in the design and construction of dams.

A. Alternatives:

(1) Spillway size and/or height of dam should be increased to pass 50 percent of the PMF. In either case, the spillway should be protected to prevent erosion.

B. O&M Procedures:

(1) Seepage and stability analyses comparable to the requirements of the recommended guidelines should be performed by an engineer experienced in the construction of dams.

(2) The minor wave erosion of the upstream embankment face should be repaired, and additional wave protection should be provided.

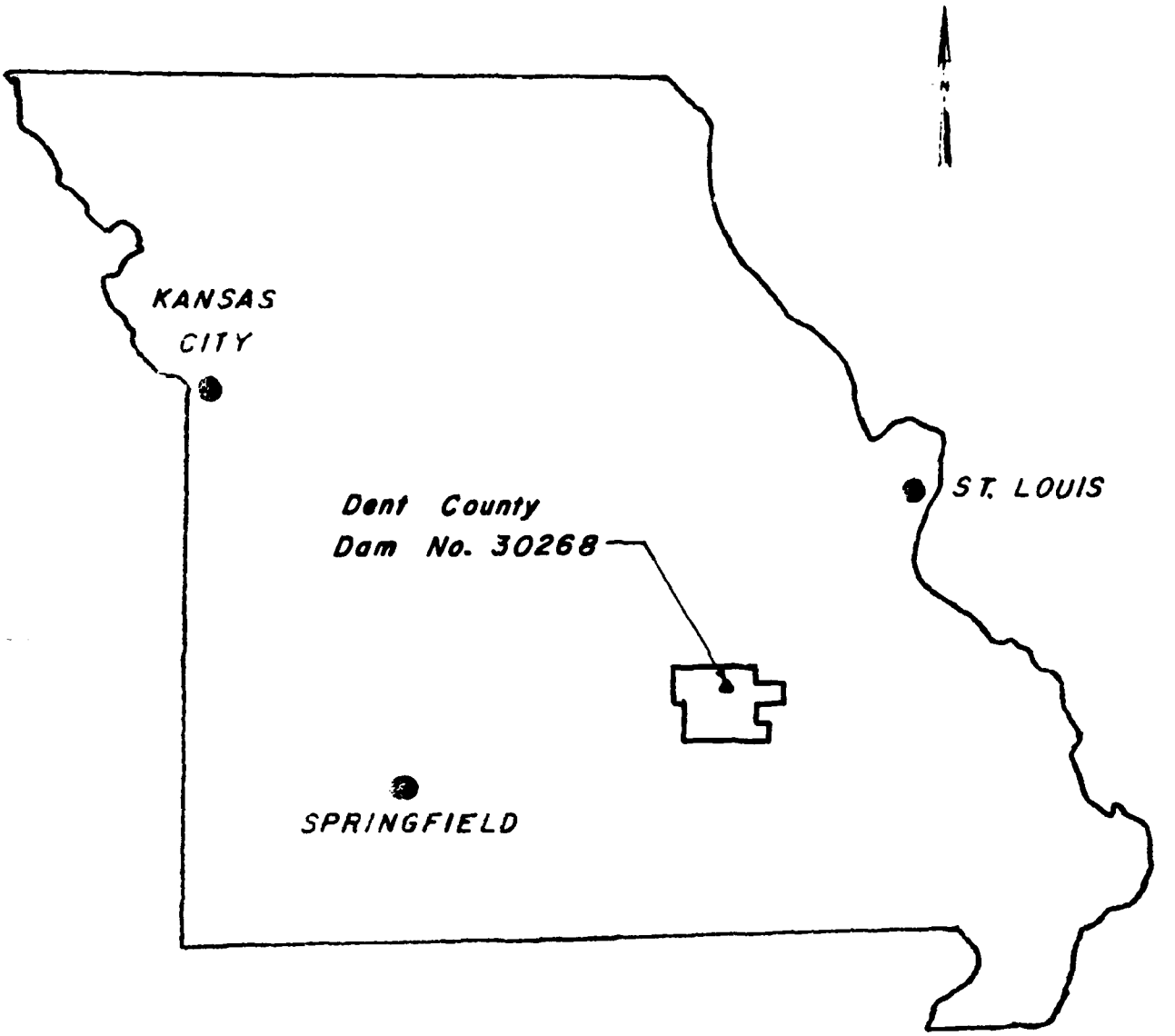
(3) The brush and weeds should be removed from the embankment face. The vegetative growth should be cut on an annual basis.

(4) The possible seepage areas at the downstream toe of the dam should be investigated by an engineer experienced in the design and construction of dams. Remedial measures may be required. As a minimum, these areas should be drained and monitored to determine if there is any increase in quantities and whether soil particles are being carried with the water.

(5) The very minor sloughing on the downstream face should be monitored and repaired if it becomes worse.

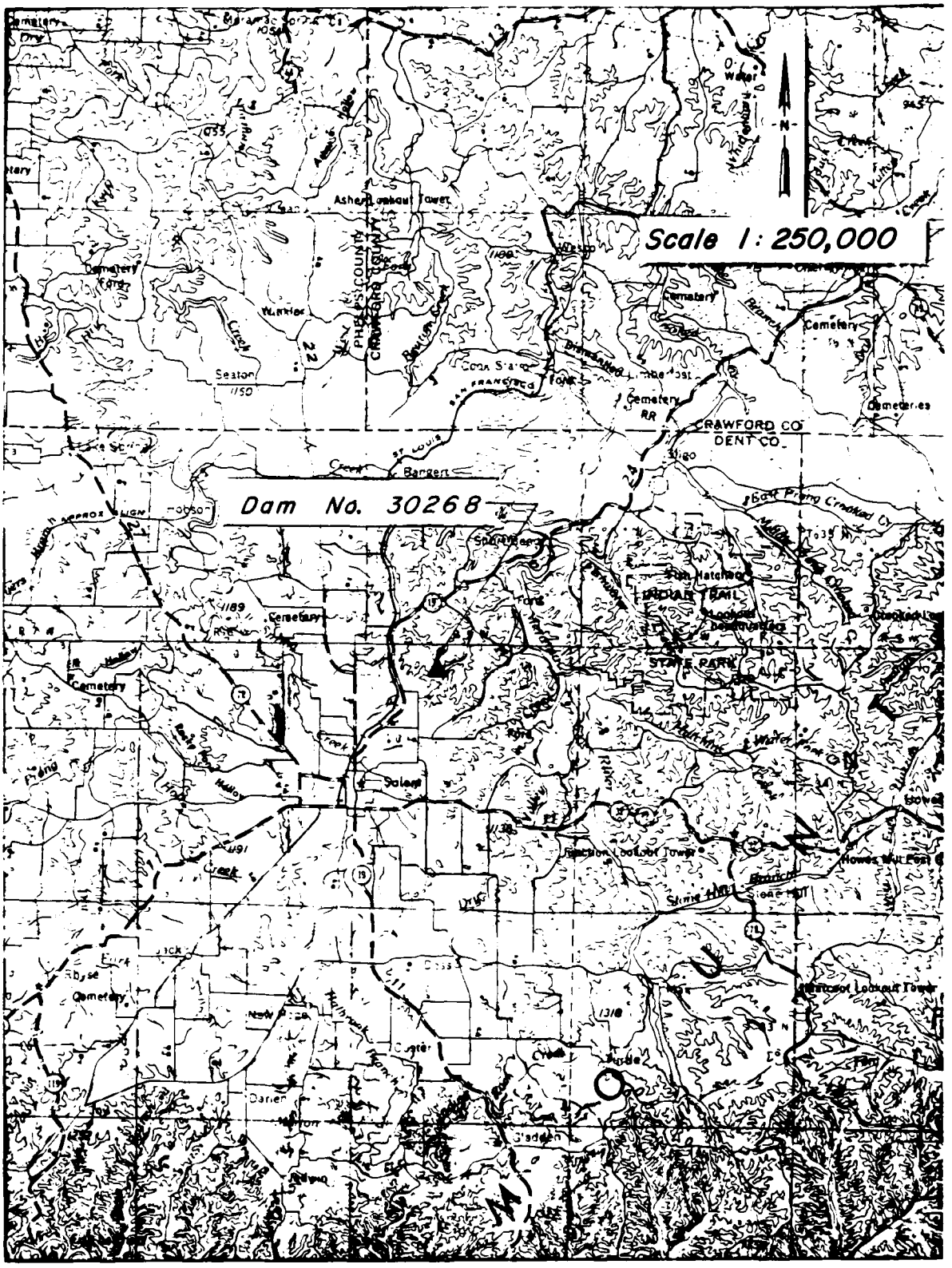
(6) A detailed inspection of the dam should be made periodically by an engineer experienced in the design and construction of dams.

APPENDIX A



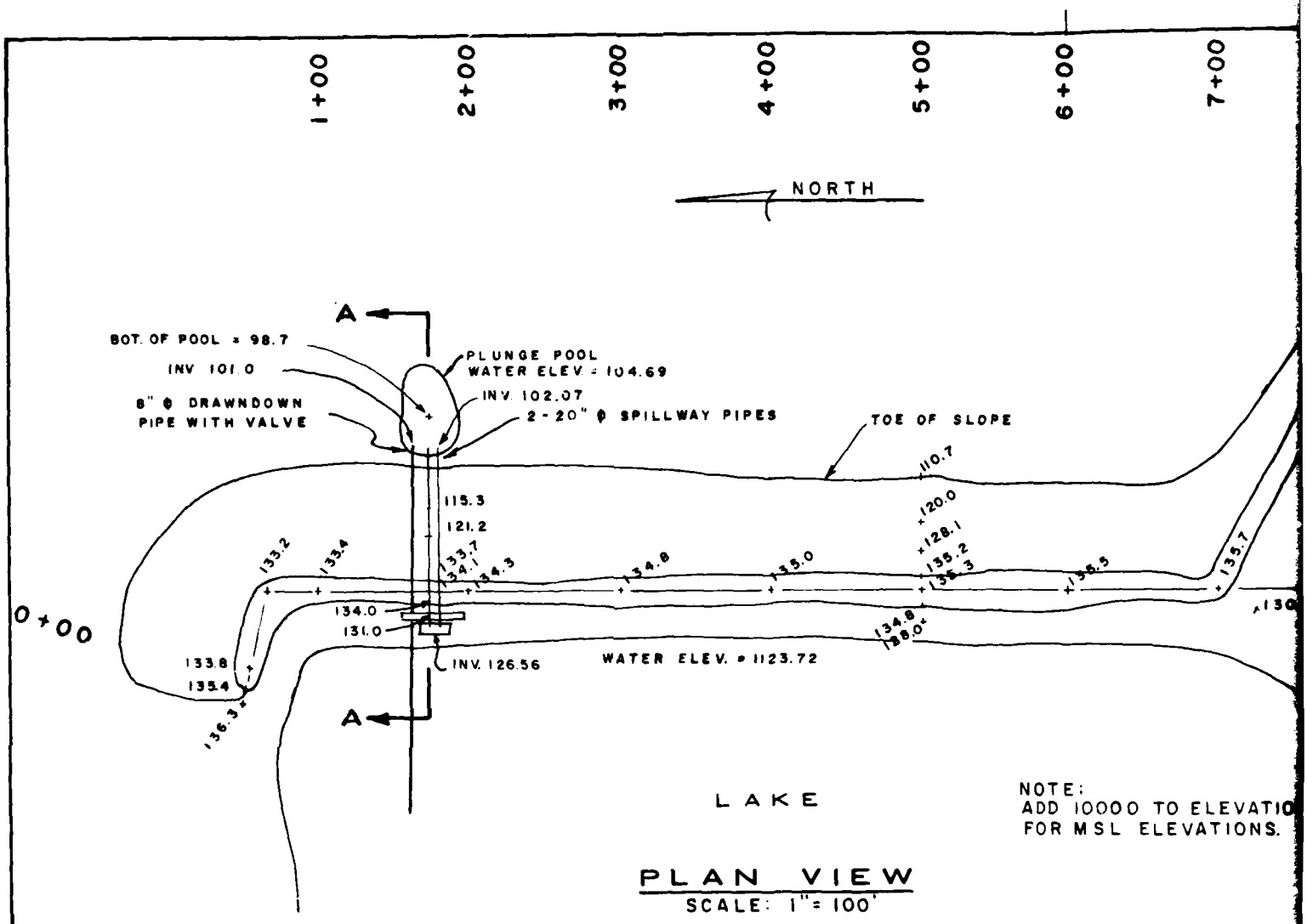
LOCATION MAP

SHEET 1 OF APPENDIX A



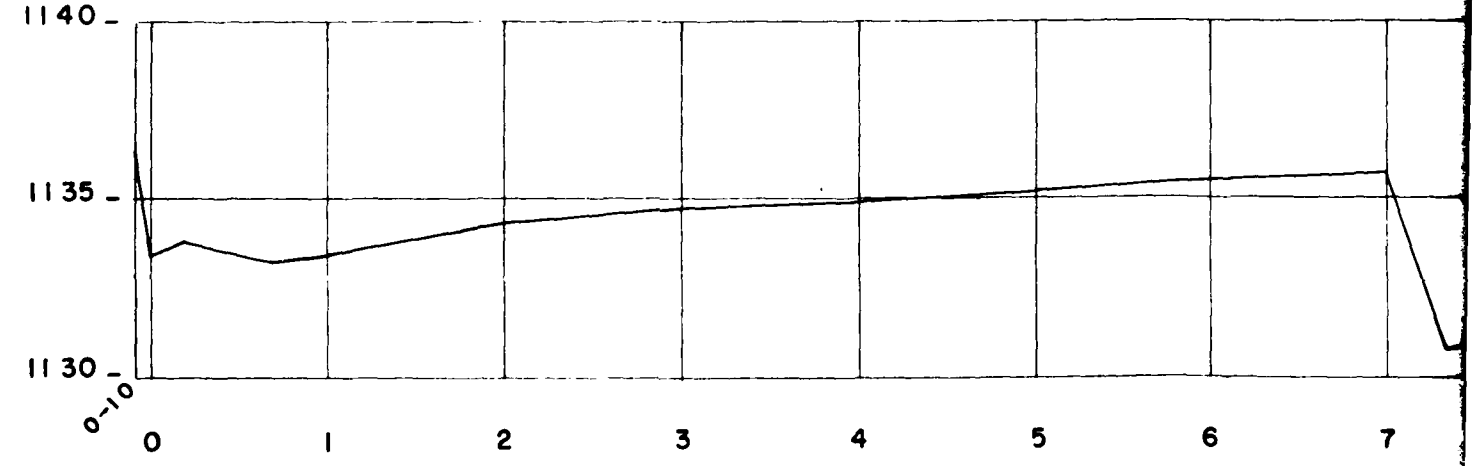
SITE VICINITY MAP

Sheet 2 Appendix A

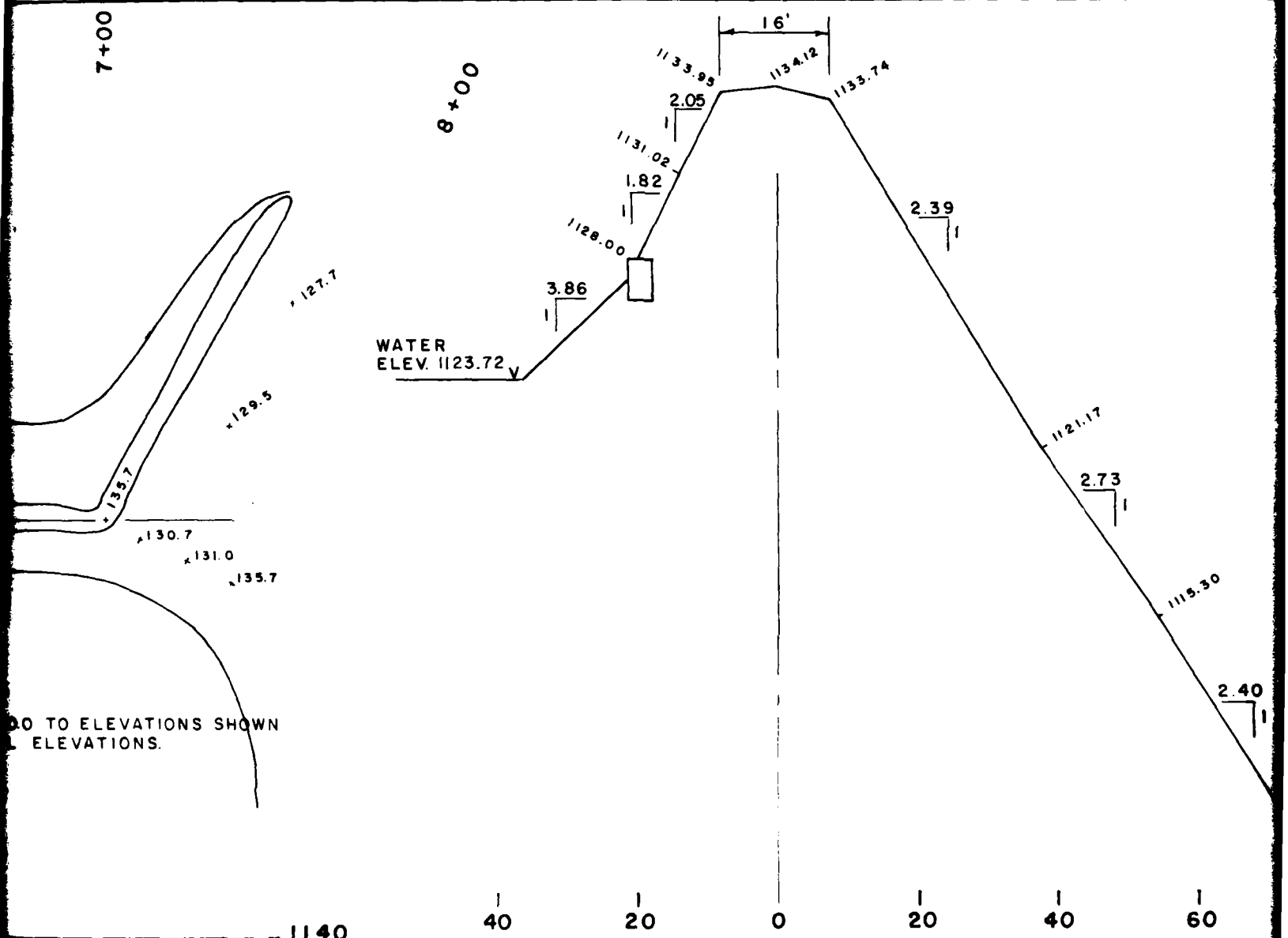


PLAN VIEW
SCALE: 1" = 100'

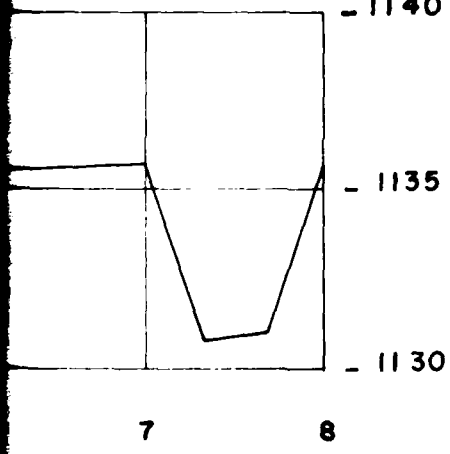
BENCHMARK:
TOP OF CREST OF STEEL WEIR FOR SPILLWAY PIPES
STA 1+75 ELEV. = 1128.00 MSL



PROFILE



SECTION A-A STA 1+75



SHEET 3 OF 4

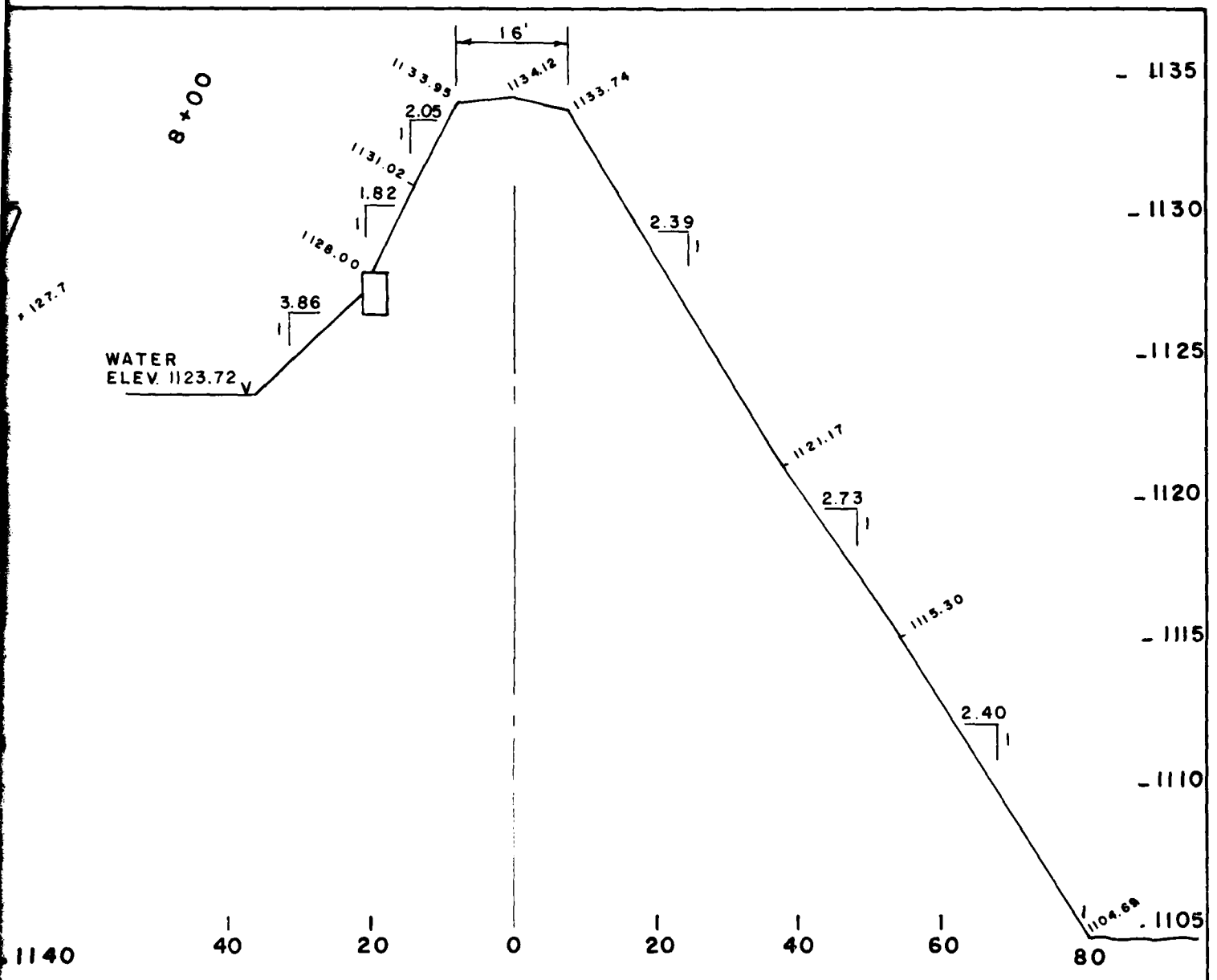
ANDERSON ENGINE
730 NORTH BENTO
SPRINGFIELD, MI

MITCHELL

MO. No. 3

PLAN & P

DENT COUN



SECTION A-A STA 1+75

SHEET 3 OF APPENDIX A

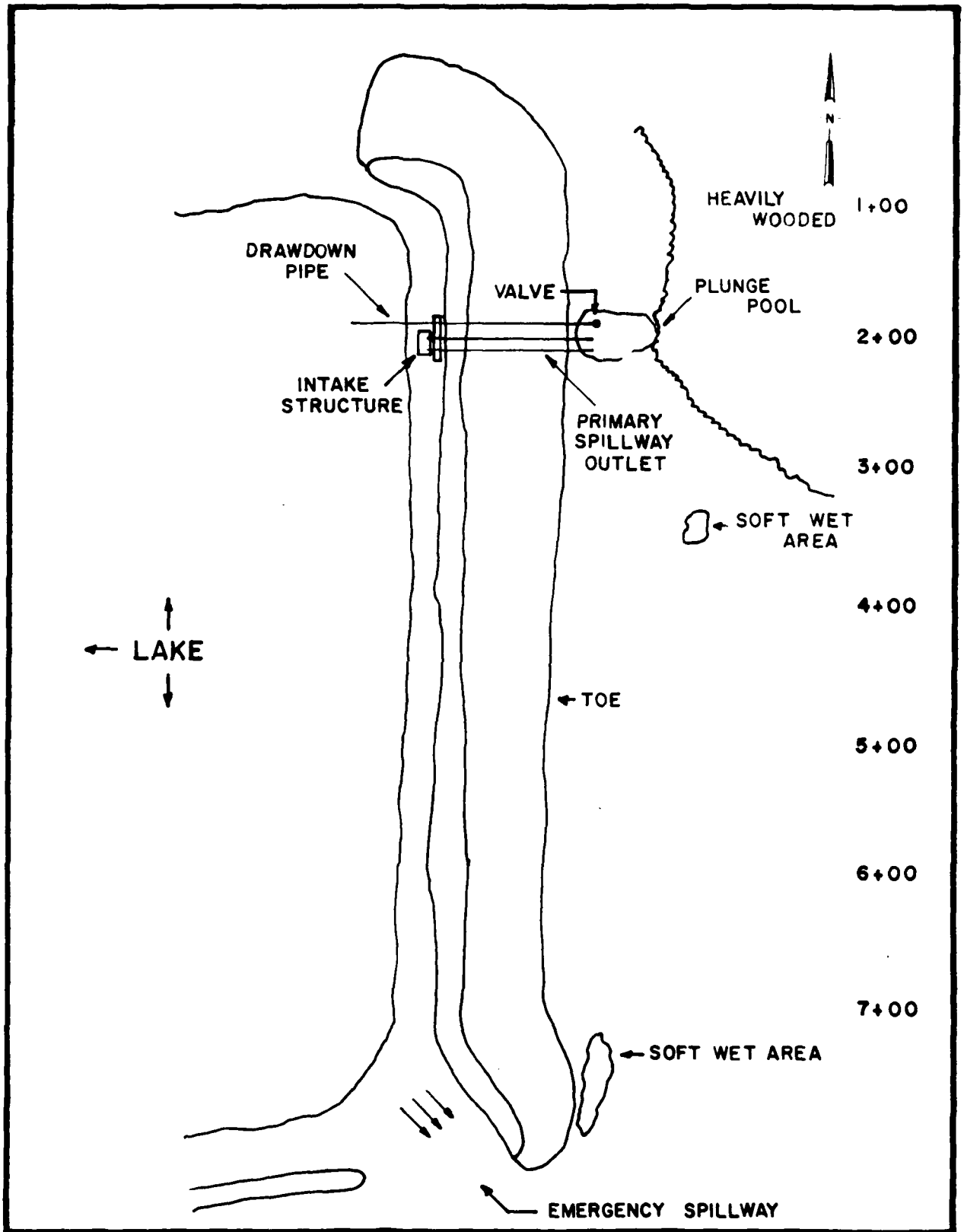
ANDERSON ENGINEERING, INC.
730 NORTH BENTON AVENUE
SPRINGFIELD, MISSOURI 65802

MITCHELL LAKE

MO. No. 30268

PLAN & PROFILE

DENT COUNTY, MO.



PLAN SKETCH OF DAM

HANSON ENGINEERS

SPRINGFIELD ILL. PEORIA ILL.

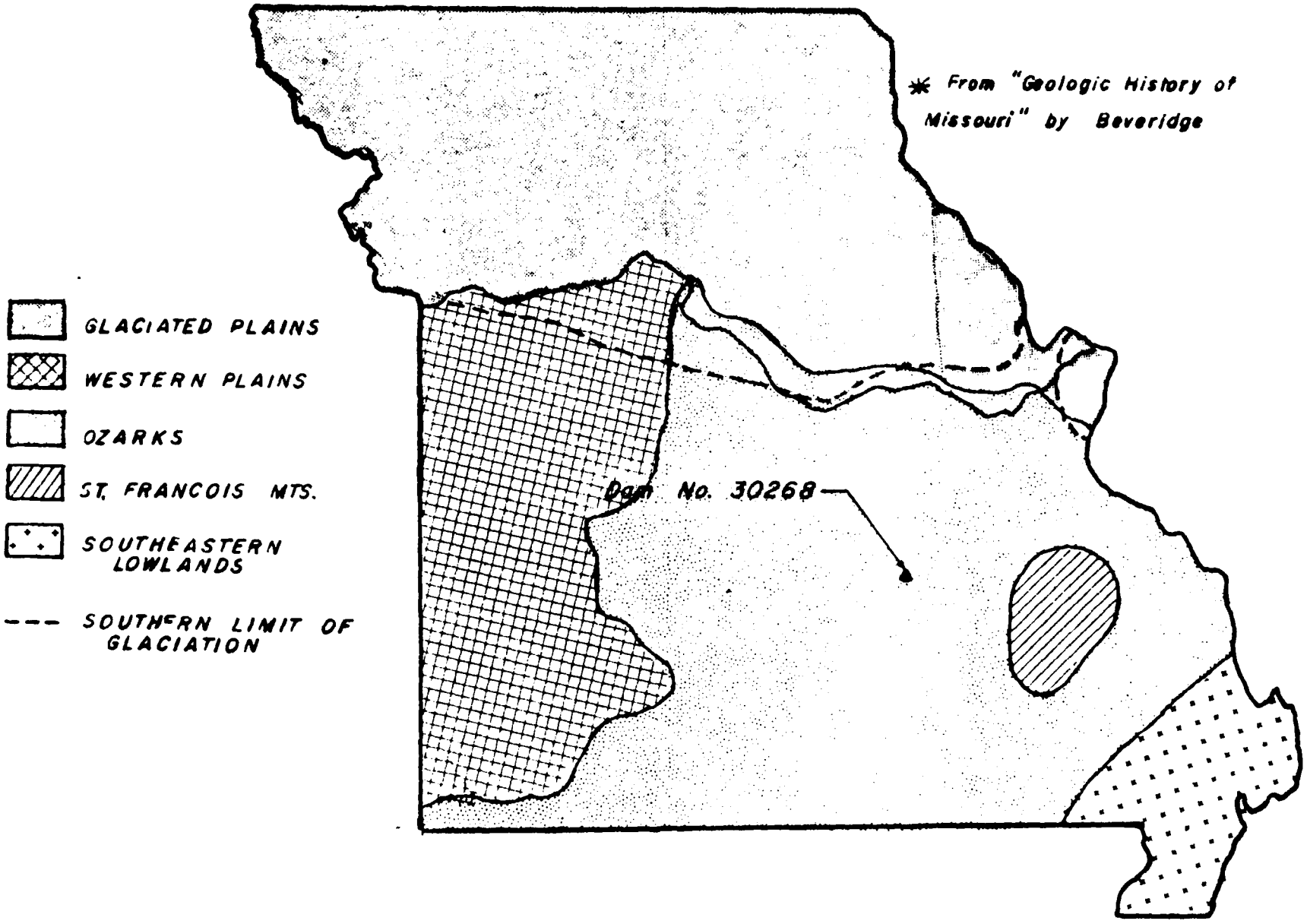
MITCHELL LAKE DAM

MO. I.D. 30268

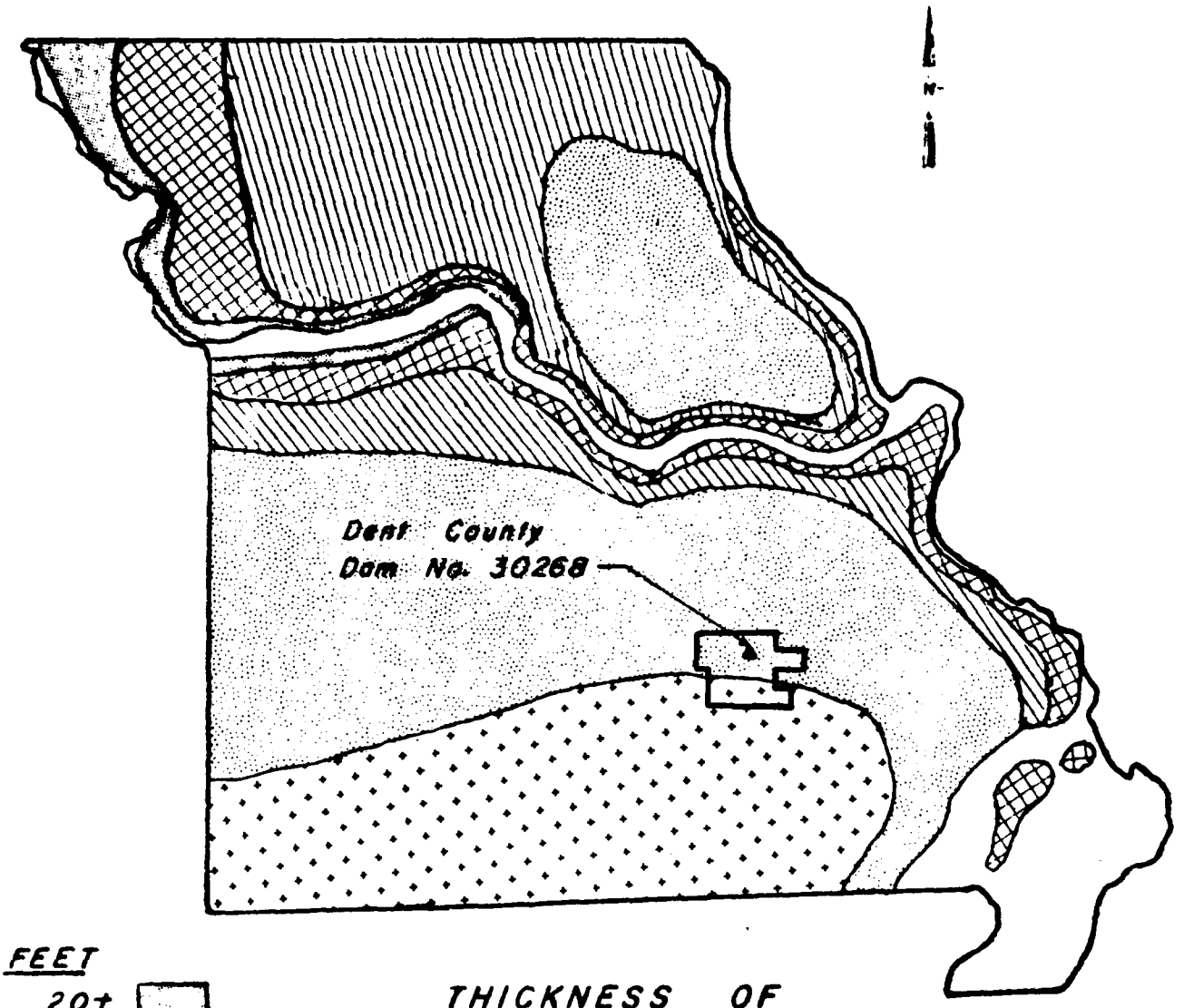
SHEET 4 APPENDIX A

APPENDIX B

MAJOR GEOLOGIC REGIONS OF MISSOURI



* From "Soils of Missouri"



FEET

20+

10-20

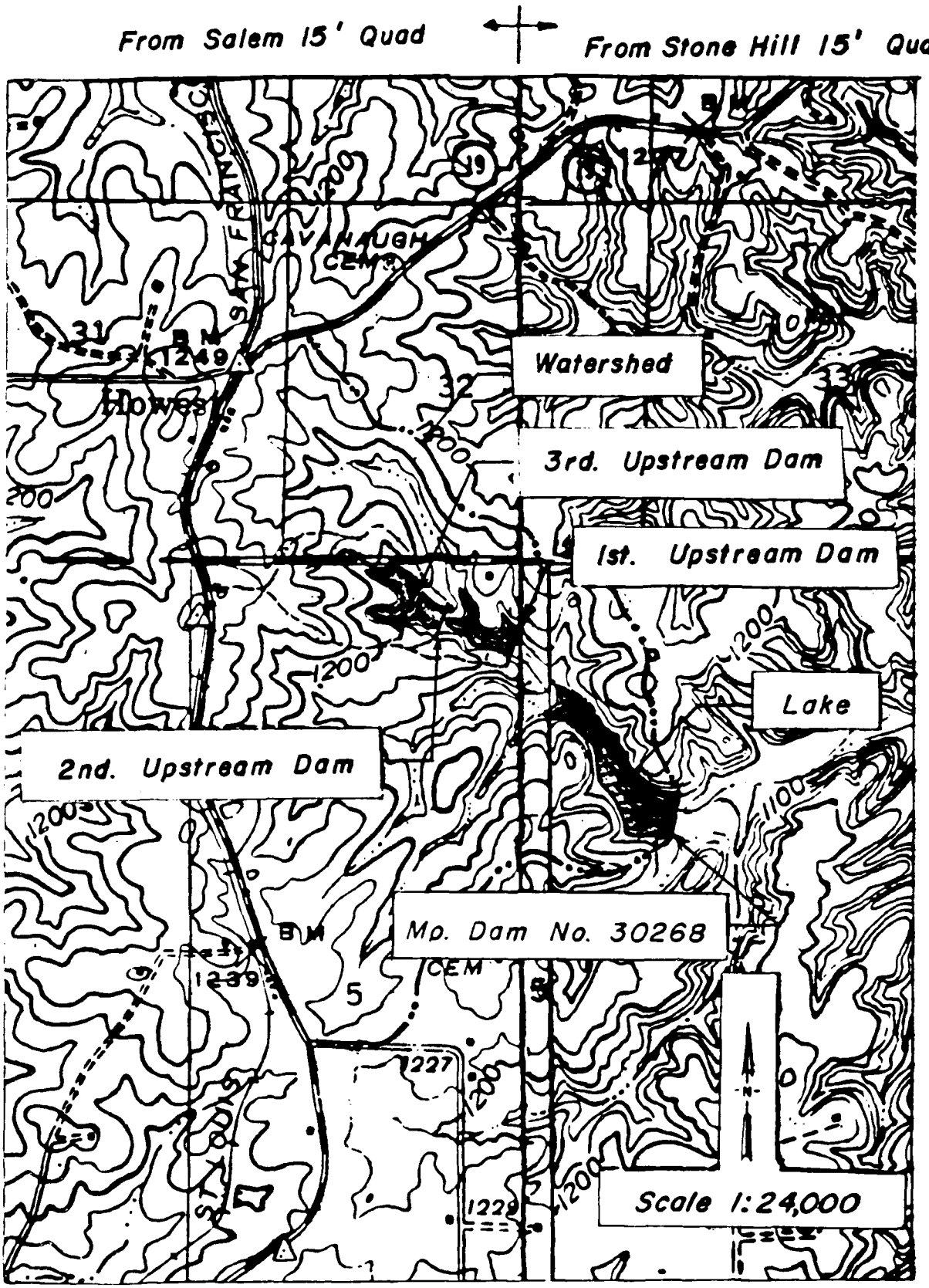
5-10

2.5-5

2.5-

THICKNESS OF LOESSIAL DEPOSITS

APPENDIX C



LAKE AND WATERSHED MAP

Sheet 1 Appendix C

HYDRAULIC AND HYDROLOGIC DATA

Design Data: From Field Measurements and Computations

Experience Data: No records are available. The owner, Dr. Roy Mitchell, indicated that the dam has never been overtopped and that after he installed the primary spillway in 1970 and enlarged the emergency spillway, all the outflows from the reservoir have been through the primary spillway. On the day of the inspection, there were no indications of high water marks or overtopping.

The owner also said that he lowers the level of the lake 5 to 6 ft every year to control the growth of algae and other freshwater plants in the shallow parts of the lake.

Visual Inspection: At the time of the inspection, the pool level was approximately 4.28 ft below normal pool (owner in process of lowering the lake level).

Overtopping Potential: Flood routings were performed to determine the overtopping potential. The watershed area was obtained by planimeter from the U.S.G.S. Stone Hill and Salem, Missouri 15 minute quadrangle map and the reservoir surface area from the U.S.G.S. Stone Hill, Missouri 7.5 minute orthophotograph (advance copy). The storage volume was developed from these data. A 5 minute interval unit graph was developed for this watershed, which resulted in a peak inflow of 1729 c.f.s. and a time to peak of 18 minutes. Application of the probable maximum precipitation minus losses results in a flood hydrograph peak inflow of 16,806 c.f.s. Rainfall distribution for the 24 hour storm was according to EM 1110-2-1411.

Based on our analyses, the combined spillways will pass 26 percent of the Probable Maximum Flood (PMF). The Probable Maximum Flood is defined as the flood discharge that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region. The recommended guidelines from the Department of the Army, Office of the Chief of Engineers, require that the structure (small size with high downstream hazard potential) pass 50 to 100 percent of the PMF, without overtopping. Considering the height of the dam (35 ft), the volume of water impounded (526 ac.-ft), the large floodplain downstream, and that the existence of three more dams upstream will reduce the effect of the design flood by storing part of the flood and by retarding the peak, 50 percent of the PMF has been determined to be the appropriate spillway design flood.

Three other dams exist upstream of this dam. To obtain a more realistic result of the flood routing studies, the PMF was considered acting simultaneously over the entire watershed area of the four dams. First, the PMF was routed through the reservoir and spillway of the third upstream dam (see LAKE AND WATERSHED MAP, Sheet 1, Appendix C). Second, the outflow hydrograph from this dam was combined with the inflow hydrograph of the second upstream dam watershed and routed through the reservoir and spillways of the second upstream dam. Third, the outflow hydrograph from this dam was combined with the inflow hydrograph of the first upstream dam watershed (MO Dam No. 30247) and routed through the reservoir and spillway of the first upstream dam. Finally, the outflow hydrograph from this dam combined with the inflow hydrograph from the watershed of the dam under consideration was routed through the reservoir and spillway of the last dam. The flood routing studies were made using the HEC-1 Dam Safety Version Program.

The routing of 50 percent of the PMF through the spillways and dams indicates that the dam under consideration will be overtopped by 1.39 ft at elevation 1136.39. The duration of the overtopping will be 1.42 hours, and the maximum outflow will be 6892 c.f.s. The maximum discharge capacity of the combined spillways is 2320 c.f.s. Analysis of the data indicates that the 100-year frequency flood will not overtop the dam. The computer input, output and hydrograph for 50 percent of the PMF are presented on Sheets 6 through 8 of Appendix C.

OVERTOPPING ANALYSIS FOR DR. ROY MITCHELL DAM

INPUT PARAMETERS

1. Unit Hydrograph - SCS Dimensionless - Flood Hydrograph Package (HEC-1); Dam Safety Version Was Used.
   Hydraulic Inputs Are As Follows:

   a. Twenty-four Hour Rainfall of 26.4 Inches For 200 Square Miles - All Season Envelope

   b. Drainage Area = 692 Acres; = 1.08 Sq. Miles

   c. Travel Time of Runoff 0.44 Hrs.; Lag Time 0.26 Hrs.

   d. Soil Conservation Service Soil Group C

   e. Soil Conservation Service Runoff Curve No. 85 (AMC III)
   70 (AMC II)

   f. Proportion of Drainage Basin Impervious 0.04

2. Spillways

   a. Primary Spillway: Two 20 in. I.D. Steel Pipes; Crest El. 1128.0

   b. Emergency Spillway: Grassed Trapezoidal Cut on Natural Earth
   Length 30 Ft.; Side Slopes 6:1 8:1; C = Varies

   c. Dam Overflow

   Length - Ft.; Crest El. 1135.0; C = Varies

3. Spillway and Dam Rating:

   Curve Prepared by Hanson Engineers. Data Provided To Computer on Y4 and Y5 Cards.
   Formula Used:

   Primary Spillway: Pipe Inlet Control Chart

   Emergency Spillway and Dam $\frac{Q^2}{g} = \frac{A^3}{T}$

Note: Time of Concentration From Equation $Tc = \left(\frac{11.9\ L^3}{H}\right)^{.385}$

California Culvert Practice, California Highways and Public Works, Sept. 1942.

SUMMARY OF DAM SAFETY ANALYSIS

1. Unit Hydrograph

   a. Peak - 1729 c.f.s.

   b. Time to Peak 18 Min.

2. Flood Routings Were Computed by the Modified Puls Method

   a. Peak Inflow

      50% PMF 8140 c.f.s.; 100% PMF 16806 c.f.s.

   b. Peak Elevation

      50% PMF 1136.39 100% PMF 1137.99

   c. Portion of PMF That Will Reach Top of Dam

      26 %; Top of Dam Elev. 1135.0 Ft.

3. Computer Input and Output Data are shown on Sheets 5 and 6 of this Appendix.

```
A       OVERTOPPING ANALYSIS FOR DR. ROY MITCHELL DAM ( # 13 )
A       STATE ID NO. 30268  CO. NO. 065  CO. NAME DENT
A       HANSON ENGINEERS INC. DAM SAFETY INSPECTION JOB # 79511
B    300                5
B1     5
J      1       7       1
J1   .15     .20     .30     .40     .50     .75     1.0
K      0       1                               3       1
K1      INFLOW HYDROGRAPH COMPUTATION FOR 2ND AND 3RD UPSTREAM DAMS
M      1       2    0.10               0.10       1                          1
P      0    26.4     102     120     130
T                                                       -1     -85             0.12
W2  0.13    0.08
X      0     -.1       2
K      1       2                           0       4       1
K1      RESERVOIR ROUTING BY MODIFIED PULS AT 2ND AND 3RD UPSTREAM DAMS
Y                               1       1
Y1     1                                                43.3      -1
Y41191.6  1192.6  1193.9    1195    1196    1198
Y5     0     139     622    1234    1974    4015
$A     0     7.6    12.3    24.8
$E1173.1  1190.2  1193.9    1200
$$1191.6
$D1193.9     3.0     1.5     670
K      0       3                               3       1
K1      INFLOW HYDROGRAPH COMPUTATION FOR 1ST UPSTREAM DAM
M      1       2    0.02               0.02       1                          1
P      0    26.4     102     120     130
T                                                       -1     -85             0.31
W2  0.04    0.02
X      0     -.1       2
K      2       3                           0       3       1
K1      COMBINE ROUTED AND LOCAL INFLOW AT IST UPSTREAM DAM
K      1       4                               4       1
K1      RESERVOIR ROUTING BY MODIFIED PULS AT 1ST UPSTREAM DAM
Y                               1       1
Y1     1                                                17.1      -1
Y41174.9    1176    1177    1178    1179    1180    1181
Y5     0     400    1161    2276    3654    5320    7310
$A     0     3.7     5.6     6.4
$E1159.2  1173.1    1178    1180
$$1174.9
$D  1178     3.0     1.5     270
K      0       5                               3       1
K1      INFLOW HYDROGRAPH COMPUTATION FOR V. HARPER DAM (#14)
M      1       2    0.43               0.43       1                          1
P      0    26.4     102     120     130
T                                                       -1     -85             0.04
W2  0.39    0.24
```

```
Y1     1                                                    17.1      -1
Y41174.9    1176    1177    1178    1179    1180    1181
Y5     0     400    1161    2276    3654    5320    7310
$A     0     3.7     5.6     6.4
$E1159.2  1173.1    1178    1180
$$1174.9
$D  1178     3.0     1.5     270
K      0       5                                  3       1
K1      INFLOW HYDROGRAPH COMPUTATION FOR V. HARPER DAM (#14)
M      1       2    0.43            0.43       1                        1
P      0    26.4     102     120     130
T                                                      -1     -85             0.04
W2  0.39    0.24
X      0      -.1       2
K      2       5                                  3       1
K1      COMBINE ROUTED AND LOCAL INFLOW AT V. HARPER DAM (# 14)
K      1       6                                  0       4       1
K1      RESERVOIR ROUTING BY MODIFIED PULS AT V. HARPER DAM (# 14)
Y                                      1       1
Y1     1                                                    78.8      -1
Y41159.2    1160    1161  1161.7    1163  1164.2    1165    1166    1168
Y5     0      33      34      35     270     826    1316    2097    4158
$A     0      11      14      23
$E1137.7  1159.2  1164.2    1180
$$1159.2
$D1164.2     3.0     1.5     375
K      0       7                                  3       1
K1      INFLOW HYDROGRAPH COMPUTATION FOR DR. ROY MITCHELL DAM (#13)
M      1       2    1.08            1.08       1                        1
P      0    26.4     102     120     130
T                                                      -1     -85             0.04
W2  0.44    0.26
X      0      -.1       2
K      2       7                                  0       3       1
K1      COMBINE ROUTED AND LOCAL INFLOW AT DR. ROY MITCHELL DAM (#13)
K      1       8                                  0       4       1
K1      RESERVOIR ROUTING BY MODIFIED PULS AT DR. ROY MITCHELL DAM (#13)
Y                                      1       1
Y1     1                                                   269.5      -1
Y4  1128    1129  1130.7    1132    1133    1134    1135  1135.7  1136.5    1137
Y5     0       6      29     198     484     970    2320    4100    7320   10260
$A     0    29.4    44.4    55.1
$E1100.5    1128    1135    1140
$$  1128
$D  1135
K     99
```

P. M. F. INPUT DATA

Sheet 6, Appendix C

********** ********** ********** ********** * ******

PEAK FLOW AND STORAGE (END OF PERIOD) SUMMARY FOR MULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
FLOWS IN CUBIC FEET PER SECOND (CUBIC METERS PER SECOND)
AREA IN SQUARE MILES (SQUARE KILOMETERS)

OPERATION	STATION	AREA	PLAN	RATIOS APPLIED TO FLOWS RATIO 1	RATIO 2	RATIO 3	RATIO 4	RATIO 5	RATIO 6	RATIO 7
				0.15	0.20	0.30	0.40	0.50	0.75	1.00
HYDROGRAPH AT	1	0.10	1	229.	305.	457.	610.	762.	1143.	1524.
	(	0.26)	(	6.47)(	8.63)(	12.95)(	17.26)(	21.58)(	32.37)(	43.16)(
ROUTED TO	2	0.10	1	67.	106.	197.	301.	399.	610.	1103.
	(	0.26)	(	1.89)(	3.00)(	5.57)(	8.53)(	11.29)(	17.26)(	31.24)(
HYDROGRAPH AT	3	0.02	1	58.	78.	117.	156.	195.	292.	389.
	(	0.05)	(	1.65)(	2.20)(	3.31)(	4.41)(	5.51)(	8.26)(	11.02)(
2 COMBINED	3	0.12	1	87.	143.	228.	353.	468.	718.	1242.
	(	0.31)	(	2.48)(	4.06)(	6.45)(	10.00)(	13.26)(	20.32)(	35.16)(
ROUTED TO	4	0.12	1	58.	112.	213.	321.	435.	695.	1107.
	(	0.31)	(	1.64)(	3.16)(	6.03)(	9.10)(	12.31)(	19.69)(	31.33)(
HYDROGRAPH AT	5	0.43	1	691.	922.	1382.	1843.	2304.	3456.	4608.
	(	1.11)	(	19.57)(	26.09)(	39.14)(	52.19)(	65.24)(	97.85)(	130.47)(
2 COMBINED	5	0.55	1	691.	994.	1576.	2137.	2699.	4143.	5703.
	(	1.42)	(	19.57)(	28.15)(	44.62)(	60.50)(	76.42)(	117.32)(	161.49)(
ROUTED TO	6	0.55	1	348.	635.	1308.	1956.	2534.	3993.	5576.
	(	1.42)	(	9.86)(	17.98)(	37.04)(	55.39)(	71.76)(	113.06)(	157.90)(
HYDROGRAPH AT	7	1.08	1	1684.	2246.	3369.	4492.	5615.	8422.	11230.
	(	2.80)	(	47.70)(	63.60)(	95.40)(	127.20)(	159.00)(	238.50)(	317.99)(
2 COMBINED	7	1.63	1	1921.	2729.	4478.	6363.	8140.	12415.	16806.
	(	4.22)	(	54.40)(	77.27)(	126.79)(	180.17)(	230.50)(	351.56)(	475.89)(
ROUTED TO	8	1.63	1	725.	1486.	3358.	5249.	6892.	11074.	14929.
	(	4.22)	(	20.54)(	42.07)(	95.09)(	148.63)(	195.15)(	313.59)(	422.74)(

SUMMARY OF DAM SAFETY ANALYSIS

3RD UPSTREAM DAM

PLAN 1 .....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1190.19	1191.60	1193.90
STORAGE	43.	55.	80.
OUTFLOW	0.	0.	622.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
0.15	1192.08	0.00	60.	67.	0.00	15.92	0.00
0.20	1192.36	0.00	63.	106.	0.00	15.92	0.00
0.30	1192.75	0.00	67.	197.	0.00	15.83	0.00
0.40	1193.04	0.00	70.	301.	0.00	15.83	0.00
0.50	1193.30	0.00	73.	399.	0.00	15.75	0.00
0.75	1193.87	0.00	79.	610.	0.00	15.75	0.00
1.00	1194.19	0.29	83.	1103.	0.33	15.75	0.00

SUMMARY OF DAM SAFETY ANALYSIS

2ND UPSTREAM DAM

PLAN 1 .....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1173.06	1174.90	1178.00
STORAGE	17.	24.	40.
OUTFLOW	0.	0.	2276.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
0.15	1175.06	0.00	25.	58.	0.00	16.67	0.00
0.20	1175.21	0.00	26.	112.	0.00	16.08	0.00
0.30	1175.49	0.00	27.	213.	0.00	15.92	0.00
0.40	1175.78	0.00	28.	321.	0.00	15.92	0.00
0.50	1176.05	0.00	30.	435.	0.00	15.83	0.00
0.75	1176.39	0.00	31.	695.	0.00	15.83	0.00
1.00	1176.93	0.00	34.	1107.	0.00	15.83	0.00

SUMMARY OF DAM SAFETY ANALYSIS

1st UPSTREAM DAM

PLAN 1 .....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1159.19	1159.20	1164.20
STORAGE	79.	79.	141.
OUTFLOW	0.	0.	826.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
0.15	1163.17	0.00	127.	348.	0.00	16.08	0.00
0.20	1163.79	0.00	135.	635.	0.00	16.08	0.00
0.30	1164.57	0.37	146.	1308.	0.50	15.92	0.00
0.40	1164.92	0.72	151.	1956.	0.83	15.92	0.00
0.50	1165.17	0.97	155.	2534.	1.17	15.92	0.00
0.75	1165.72	1.52	163.	3993.	3.58	15.83	0.00
1.00	1166.18	1.98	170.	5576.	5.08	15.83	0.00

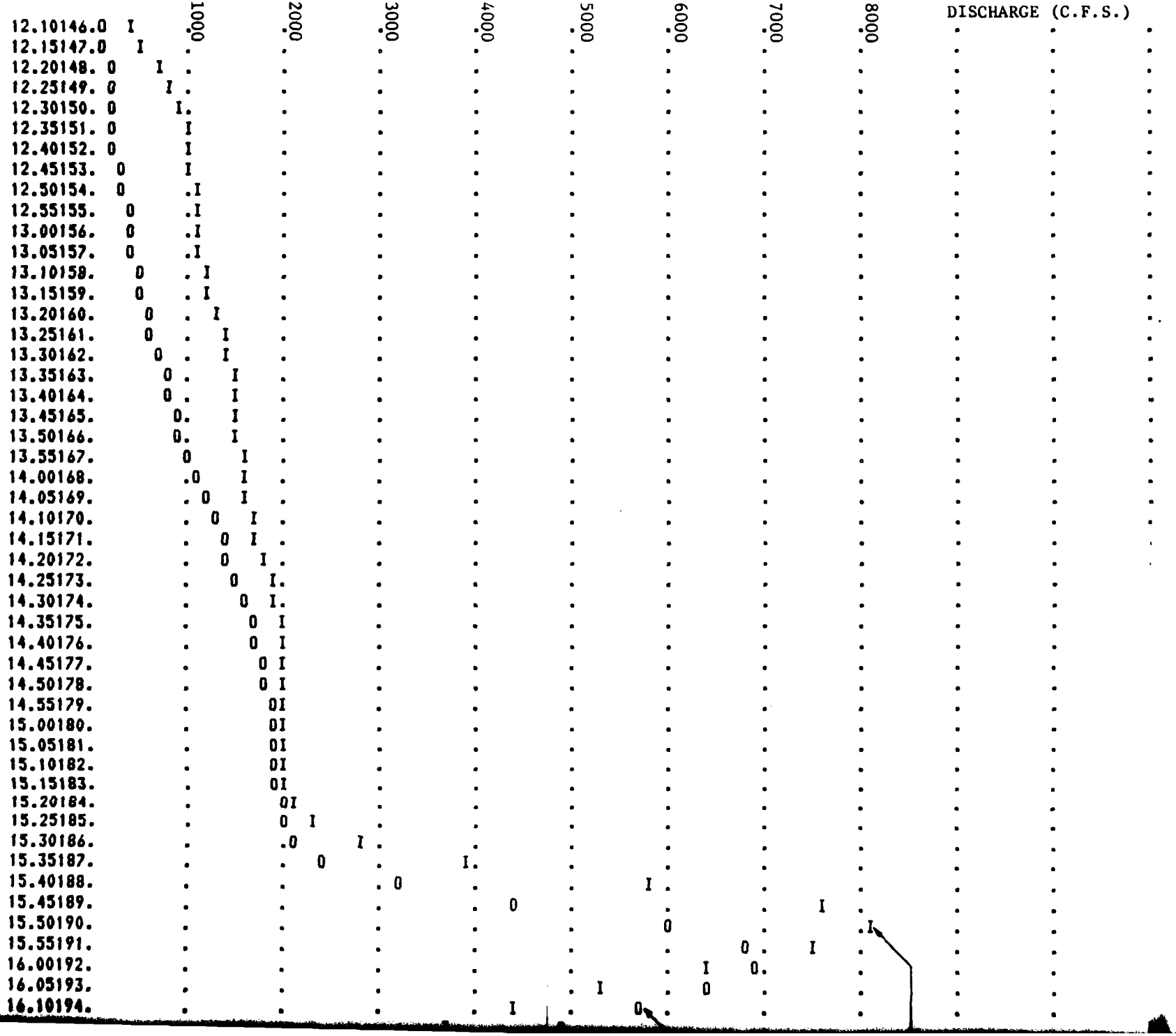
SUMMARY OF DAM SAFETY ANALYSIS

DR. ROY MITCHELL DAM

PLAN 1 .....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1128.00	1128.00	1135.00
STORAGE	269.	269.	526.
OUTFLOW	0.	0.	2320.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
0.15	1133.50	0.00	462.	725.	0.00	16.58	0.00
0.20	1134.38	0.00	499.	1486.	0.00	16.25	0.00
0.30	1135.41	0.41	544.	3358.	0.58	16.08	0.00
0.40	1135.99	0.99	571.	5249.	1.00	16.00	0.00
0.50	1136.39	1.39	590.	6892.	1.42	16.00	0.00
0.75	1137.17	2.17	627.	11074.	4.00	15.92	0.00
1.00	1137.99	2.99	668.	14929.	5.17	15.92	0.00


1000
2000
3000
4000
5000
6000
7000
8000
DISCHARGE (C.F.S.)
12.10146.0
12.15147.0
12.20148.0
12.25149.0
12.30150.0
12.35151.0
12.40152.0
12.45153.0
12.50154.0
12.55155.0
13.00156.0
13.05157.0
13.10158.0
13.15159.0
13.20160.0
13.25161.0
13.30162.0
13.35163.0
13.40164.0
13.45165.0
13.50166.0
13.55167.0
14.00168.0
14.05169.0
14.10170.0
14.15171.0
14.20172.0
14.25173.0
14.30174.0
14.35175.0
14.40176.0
14.45177.0
14.50178.0
14.55179.0
15.00180.0
15.05181.0
15.10182.0
15.15183.0
15.20184.0
15.25185.0
15.30186.0
15.35187.0
15.40188.0
15.45189.0
15.50190.0
15.55191.0
16.00192.0
16.05193.0
16.10194.0

INFLOW-OUTFLOW

HYDROGRAPH

FOR 50% P. M. F.

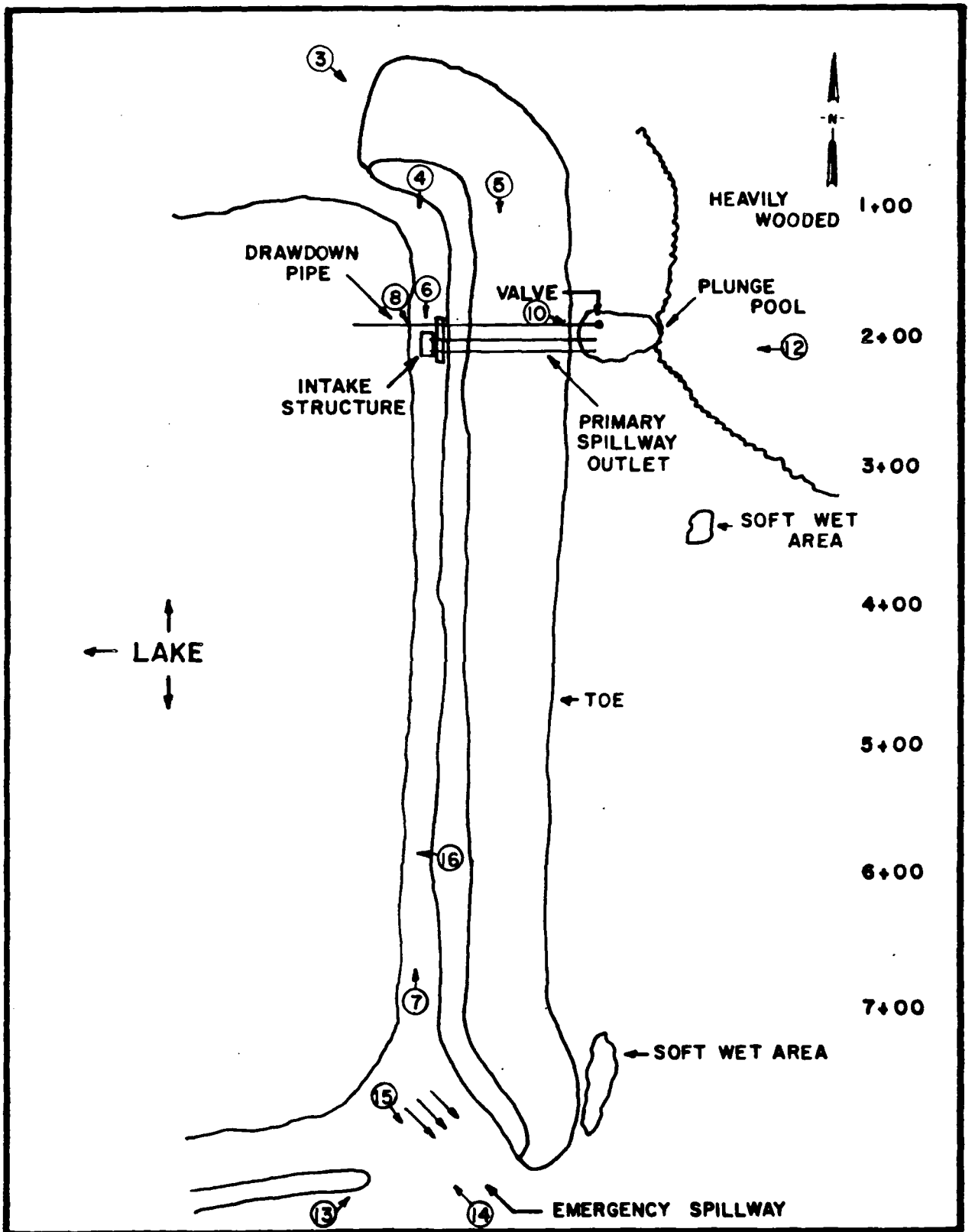
MAX. INFLOW = 8140 C. F. S.

MAX. OUTFLOW = 6892 C. F. S.

APPENDIX D

INDEX TO PHOTOGRAPHS

Photo No.	Description
1	Aerial - Lake and Dam, Looking Southwest
2	Aerial - Lake and Dam, Looking West
3	Overall View of Dam, Looking South
4	Crest of Dam, Looking South
5	Downstream Face of Dam, Looking South
6	Upstream Face of Dam, Looking South
7	Upstream Face of Dam, Looking North
8	Primary Spillway Intake Structure
9	Close-Up of Inlet and Anti-Vortex Wall
10	Primary Spillway Outlet and Plunge Pool
11	Close-Up of Primary Spillway Outlet (Note Valve Wheel)
12	Primary Spillway Discharge Channel, Looking Upstream
13	Emergency Spillway Area, Looking Northeast
14	Emergency Spillway, Looking Upstream
15	Emergency Spillway, Looking Downstream
16	View of Lake and Watershed



PLAN SKETCH OF DAM

HANSON ENGINEERS

SPRINGFIELD ILL. PEORIA ILL.

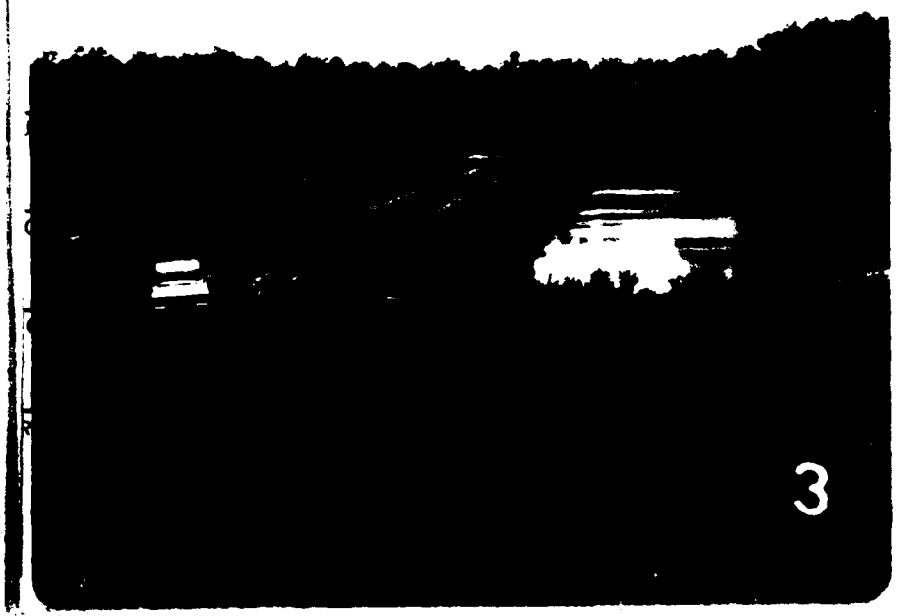
MITCHELL LAKE DAM

MO. I.D. 30268

SHEET 2 APPENDIX D


1


2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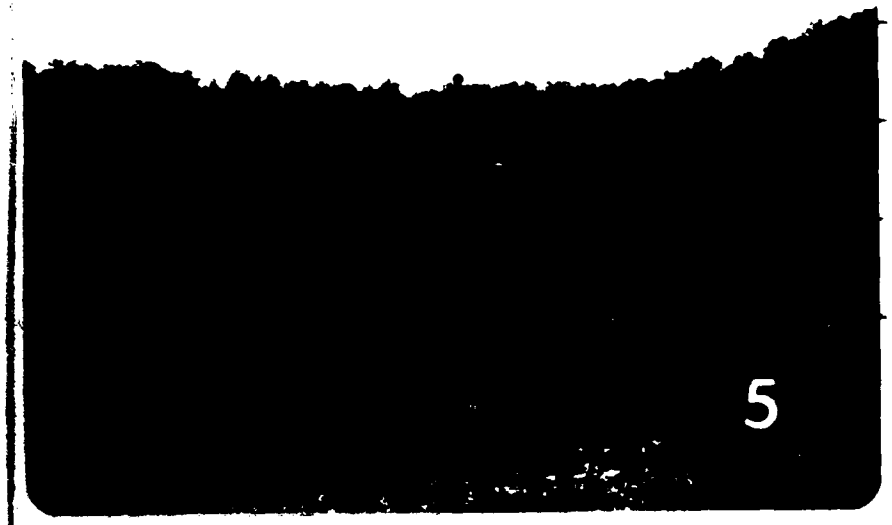
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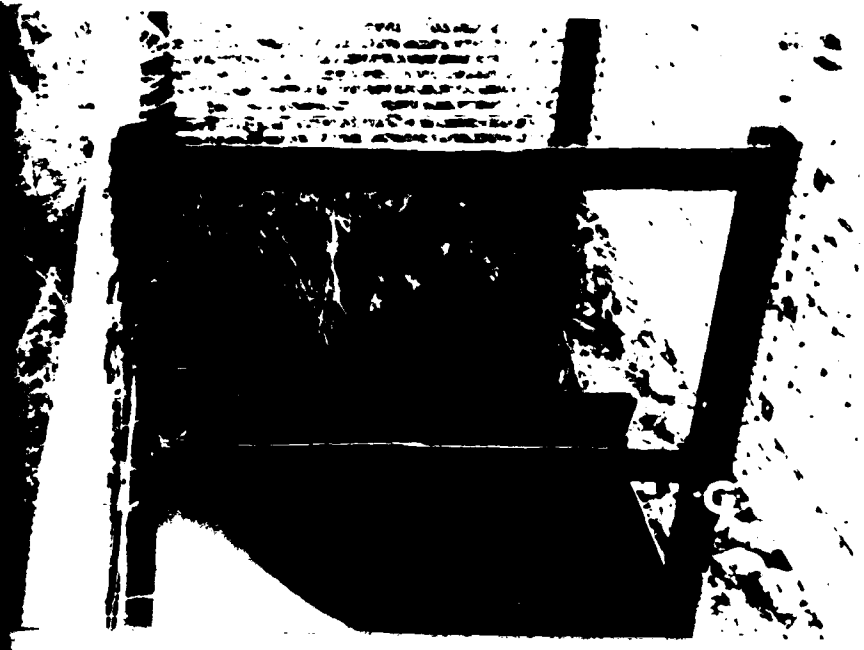


12


13


14


15


16


DAT
ILM